Fukushima Daiichi Nuclear Power Plant: A Retrospective Evaluation

P. Carydis
Professor Emeritus, National Technical University of Athens, Athens, Greece

A. Pomonis
Cambridge Architectural Research Ltd, United Kingdom

K. Goda
University of Bristol, Department of Civil Engineering, United Kingdom

SUMMARY:
Based on available data and personal experience in Japan at the time of the design and construction of the Fukushima Daiichi nuclear power plant (NPP), the basis of its earthquake resistant design is revisited. The relevant knowledge and technology of the time (late 1960's to early 1970's) are reviewed to illustrate the technical environment in which this and other plants around the world were designed. The initial structural design of the plant was based on static analysis and for structures and components needing high seismic protection, a seismic coefficient of 0.54 was used. A comparison is carried out between the design seismic loads and the inferred seismic excitations that were likely to be experienced at the Fukushima Daiichi NPP during the 11th March 2011 earthquake. It is indicated that the allowable checking stress reached the yielding material stress. The piping system and the containment structure were dynamically designed based on a pre-described response spectrum shape with peak ground acceleration of0.27g. It is concluded that structural components' upgrading should be as much as around three times the initial seismic resistance in order for the plant to safely withstand the 2011 ground motion.

Keywords: March 11th 2011 Tōhoku earthquake, Fukushima Daiichi nuclear power plant, seismic design

1. INTRODUCTION

On Friday 11th March, 2011, at 14:46 Japan Standard Time, a M_w9.0 earthquake occurred at the Japan Trench off the coast of Tōhoku in north-east Japan. Ground shaking was felt as far as western Japan and lasted for almost four minutes (220 sec), generating large, unprecedented tsunami waves that caused the loss of around 19,200 lives (incl. circa 3,000 people missing). This M_w9.0 earthquake is the largest event that has been recorded in Japan since the beginning of instrumental seismology circa 1900. The Fukushima Daiichi nuclear power plant (NPP) is situated on the Pacific coast of northern Honshu Island near Okuma Town and Futaba Town, Futaba County, Fukushima Prefecture and consists of six reactors that commenced operation between 1971 and 1979 (IAEA, 2011a). There were three reactors, one, two and three, operating at the time when the 2011 earthquake occurred, while reactors four, five and six had been shut down as part of routine maintenance work. The strong ground shaking triggered the immediate shut down of the three operating reactors. The control rods were automatically inserted into the reactors, halting the fission reaction that generates electricity. The same shutdown process happened at other eight reactors in Japan (EEFIT, 2011), causing a sudden loss of power across Japan's national power grid, widespread power failures and cutting vital electricity supplies to Fukushima Daiichi. In addition there was seismic damage to the external grid infrastructure that resulted in the loss of external power supplies (EEFIT, 2011). This was critical because when it is shut down, it is reliant on the external grid, on-site emergency diesel generators, or other reactors at site for AC power (ONR, 2011). The design tsunami height in Fukushima Daiichi was 5.4 to 5.7 m (JSCE, 2002) and to include a margin of safety the elevation of the plant was set higher (IAEA, 2011b) – the site elevation of Fukushima Daiichi NPP is 10 m at Units 1 to 4, and 13 m at Units 5 and 6 (EEFIT, 2011). About 45 minutes after the earthquake, 14 to 15 m high tsunami waves overpowered the plant's sea protection wall and flooded much of the facility (TEPCO, 2011), disabling the critical back-up power generators that were placed at elevation lower than the tsunami flood zone.

2. HISTORICAL BACKGROUND

The main author, from the end of 1968 to the middle of 1970 attended the advanced course of the International Institute of Seismology an Earthquake Engineering (IISEE), in Tokyo, carrying out post-doctoral research and teaching. In the late 1960's at the IISEE, a major activity related to the huge Japanese project for constructing more than 50 NPPs was in progress. Similar projects were undertaken around the globe at the same period. Japan had already established working groups with the United States in order to set common design rules and guidelines for earthquake design and construction of NPPs.

As an assistant that time to Prof. M. Watabe, the main author followed the relevant proceedings and participated to the respective meetings of the Japanese and foreign experts in seismology and earthquake engineering. Due to his expertise in analogue computation, he conducted extensive parametric dynamic analyses of mathematical models of NPP components, which had been already designed (some were in the construction phase and others were in the design phase). This was done to assess the earthquake performance of those components by changing various parameters, mainly for decision-making. All basic components including soil-structure interaction were considered. The proceedings of one of the above mentioned meetings are contained in a volume entitled "Proceedings of the IAEA Panel on Aseismic Design and Testing of Nuclear Facilities" held on 12th-16th June 1967 in Tokyo, and published by The Japan Earthquake Engineering Promotion Society in 1969 (IAEA, 1969). This rather important document underpins most of the arguments in the present paper.

A NPP installation is a quite complex structure with its various components dynamically interacting with each other – one structure is inside the other as the Russian dolls "babushka". The connecting parts are extremely vital for the function and the safety of the whole installation. Therefore, it was a basic requirement to carry out an integrated structural analysis for the statically acting forces (including thermal and gas pressure actions) combined with the forces resulting from the dynamically acting earthquake motions for the whole structural system including various equipments attached to it in various ways and connecting various members of the plant.

At that time, it was nearly impossible to carry out such sophisticated computational analyses. Instead, more rough and simplified approaches of analysis were applied in order to get practical solutions to the problems that were approximated based on engineering judgment and experience. The methodology was founded on the analysis of standard structures. Typically, the design logic accepts that minor cracks will incur, during the action of the design earthquake, while in case of stronger shaking, all codes admit greater cracks and damage to occur (but in any case no collapse). However, the creation of cracks in a NPP structure and its components must be by no means permissible. Every part of the structural system must behave linearly and elastically, meaning that after the occurrence of the design earthquake the structure returns to the state it was before. Hisada (1967) stated that: "The allowable tensile unit stress for the structural steel is taken as nearly yield stress. The allowable compressive unit stress for concrete is two-thirds of ultimate compressive strength at 28 days. When secondary, local and peak stresses are additionally considered, the computed stress, is permitted to exceed the yield stress provided that excessive strains will not be incurred". It must be emphasized that the allowable stresses for permanent loads are about two-thirds of the steel yielding stress and one-third of the ultimate concrete compressive strength at 28 days. This assumption is not even valid for common structures, in which, for the earthquake load combination, the increase of the allowable stresses is no more than 20 to 25%. It is well known that when we enter the yielding level of materials, cracks and non-linearity start to dominate the dynamic response with increasing deformations since the phenomenon is accumulative.

For common structures, the concept of ductility was invented in order to design structures with reduced earthquake accelerations, than those that actually excite them during strong earthquakes. To ensure the safety of the structure when using the ductility (capacity) factor higher than 1.0, the structure must be capable of withstanding cyclic displacements in the plastic region without loosing its strength, stiffness and energy dissipation ability. In the late 1960's, the ductility concept and the relevant parameters were at the dawn of their development, without possessing any qualitative or quantitative expression to be used in structural design practice. However, detailed knowledge of the seismic response of structures and materials simultaneously under high temperatures and pressure, required to control the development of major cracks in NPP components, was lacking (Housner, 1967). In this context the problems of the dynamic interaction with the foundation ground and the behaviour of the ground materials on which the massive and stiff structures of a NPP are founded, were still under investigation and much research work was still needed.

The recommendations for the foundation and structures of NPPs compiled at that time by Hisada (1967) –but without any legal status– stated that: "The structures are constructed on firm ground. The important structures, such as the reactor building, are usually supported (directly or through rigid foundations) on bedrock or equivalent layers. Special attention is given to avoid the differential settlement of the foundation. Buildings and structures in NPPs are designed to be as rigid as possible. A reactor building is made of monolithic concrete construction which has sufficient strength, rigidity and ductility as a whole".

The concept of using seismic base isolation, of any type, and of other seismic energy absorbing devices was still under investigation at that time. As a result any earthquake motion equal to or exceeding the design level, would almost certainly cause damage to the structure. This was the case, and still continues to be, when there are no additional mechanisms or other provisions to absorb the seismic energy in the case of a strong earthquake. In the latter case, the structure absorbs any excessive input seismic energy by creating large deformation into the nonlinear domain. To the best of our knowledge, dampers and bellows were used only at structural supports of the piping system (Fig. 1), while a stabilizer was used around the head of the reactor pressure vessel (RPV), containment and shield wall structures (Fig. 2).

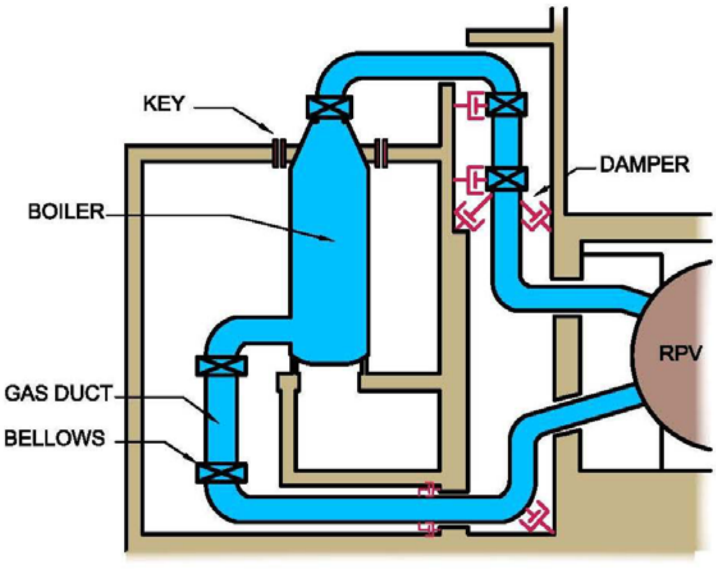

Figure 1. Bellows, dampers and keys provided at critical points of the primary piping circuit by courtesy Watabe (1970).

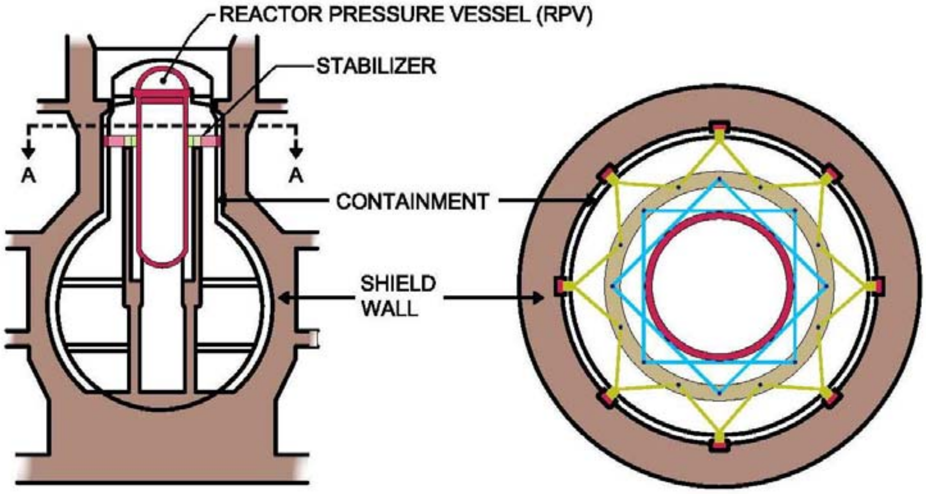

Figure 2. Stabilizing and impact protecting system among RPV, containment and shield wall structure by courtesy Watabe (1970).

The problems involved with this great endeavor – internationally – were not only on the domain of computational analysis and structural design mentioned above, but also on seismic hazard estimation. From the seismological point of view, significant amount of new findings have been discovered since 1970's (e.g. potential seismic sources near a NPP site and their characteristics for generating strong ground shaking). Unfortunately, such information was not available at the time when the Fukushima Daiichi NPP was designed and constructed. To overcome this deficiency, existing design guidelines at that time recommended to follow the respective building code seismic zones with base seismic coefficients according to the classification of various components of the plant, based on their importance. In Japan, the seismic zonation map was revised when the 1984 Japan seismic code was introduced. The Fukushima Daiichi plant, when designed belonged to the moderate seismicity zone (B), with a base seismic coefficient of 0.18 but in 1984 the assignment was changed to the highest seismicity zone (A), having a seismic coefficient of 0.20 (Japanese Ministry of Construction, 1984). The need thus had aroused that many old structures although properly designed and constructed besides the necessary maintenance should be seismically checked and, if necessary, upgraded according to new seismic provisions. The importance of checking/upgrading was much higher for critical facilities. Of equal importance is the need to determine the characteristics of a probable tsunami that may affect nuclear installations along coastlines, although this could be deemed as unnecessary, since, in the recommendations (IAEA, 1969), it is clearly stated that: "to select a site having no danger of tsunamis" (Hisada, 1967).

In conclusion, in the late 1960's and beginning of 1970's, the existing knowledge was not sufficient in order to ensure the reliability and safety of NPPs around the world. As seismology, earthquake engineering and material science have made outstanding progress during the decades following the design and construction of those NPPs, the respective scientific and technical achievements should have been incorporated time to time. The Fukushima Daiichi NPP is one representative case of any plant built not only in that period of time but also several years later.

3. DESIGN ANALYSIS

Based on the afore-mentioned expert meeting proceedings, peak ground accelerations at the safe shutdown earthquake (SSE) and the maximum credible earthquake (MCE) levels for the seismic design of several NPPs in the late 1960's are summarized in Fig. 3.

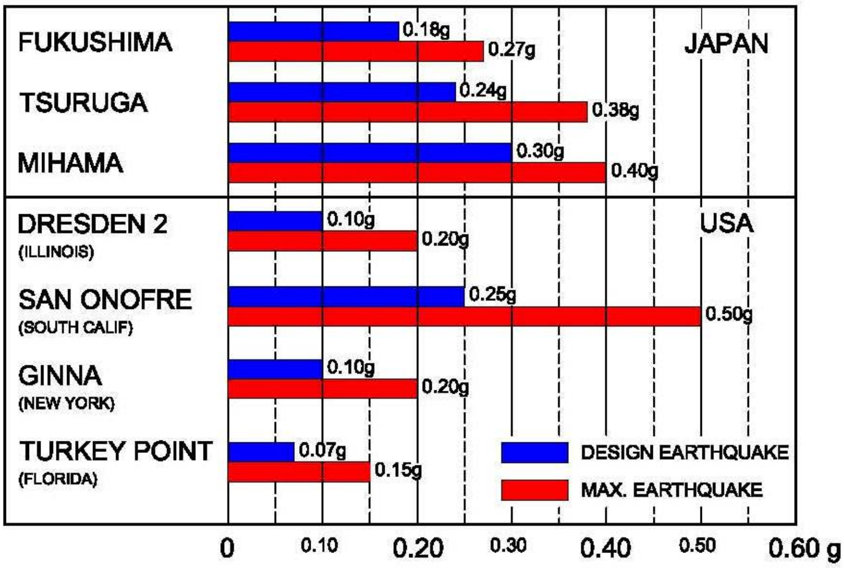

Figure 3. Ground accelerations corresponding to safe shutdown earthquake (SSE) and maximum credible earthquake (MCE) for several NPPs constructed at the time under consideration (Morris, 1967) and by courtesy Watabe (1970).

The terms SSE and MCE refer to the most probable and the least probable earthquake motions to occur at the site, respectively. Those probabilistic aspects are closely related to the nominal life span of the structure, and to the acceptable margins of safety. The SSE is also called as operating basis earthquake (OBE).

The design forces and displacements are calculated according to a pre-described response spectrum, a curve from which the maximum accelerations developed in a structure may be calculated according to its natural period and damping characteristics. To our best estimates, the Fukushima Daiichi plant in its undamaged state was a quite stiff structure with its members (core, reactor pressure vessel, containment and reinforced concrete exterior building) having natural periods up to 0.50 sec. For these natural periods, Daiichi's components, with damping ratios of about 2–5%, were designed according to the following assumptions: NPP components are classified into four classes, As, A, B, and C, according to their importance from the overall safety viewpoint either under normal operating conditions or during a strong earthquake. For each class, the respective design parameters were defined. A brief presentation of the earthquake design parameters for the basic components of the Fukushima Daiichi NPP is given in Fig. 4.

CLASS As. Includes structures, equipment and piping system which are especially vital for the safety of the NPP and their function must be maintained under a larger earthquake action than the design level. These items are the containment structure, the primary cooling system and the gas ducts. The seismic loads for the design are calculated based on dynamic analysis using a pre-described response spectrum (see Fig. 5) with a ground acceleration corresponding to the MCE, which for the case of the Fukushima Daiichi NPP was 0.27g.

CLASS A. Includes items that are vital and whose functional loss might cause a serious nuclear accident, or items under the normal function of which the environment is protected from radiation. Structures in this class are the reactor building, the containment structure, the control room, while the respective equipment and piping system are the reactor, the safety shutdown device, the primary cooling system, the emergency cooling system, and the emergency power supply system. The design horizontal seismic coefficient (c_h) was equal to three times the yielding at the respective seismic zone of the Japanese Building Code, i.e. $c_h = 3\times0.18 = 0.54$. For the vertical seismic loads, $c_v = 1/2\times0.18 = 0.09$. Items in class As were designed dynamically and statically belonging also to class A, and the safer was chosen.

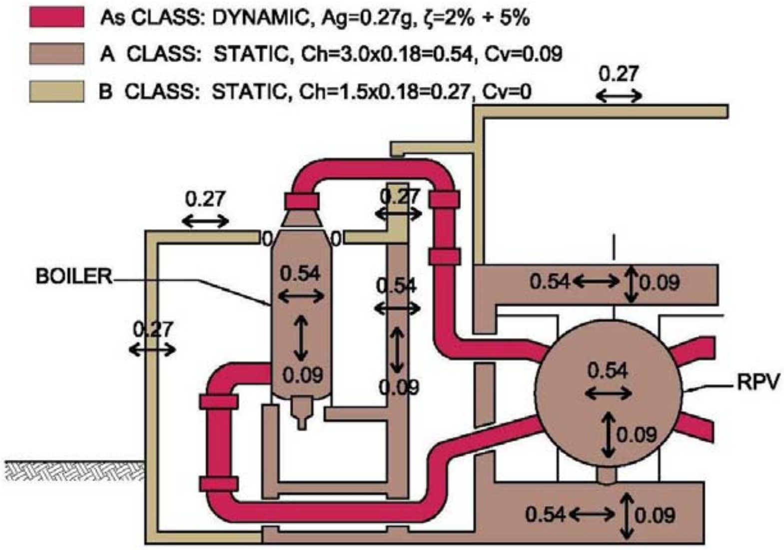

Figure 4. Basic seismic design parameters for the Fukushima Daiichi NPP. The checking stresses are at about the yielding material strength level, (Hisada, 1967) and by courtesy Watabe (1970).

CLASS B. Includes items that require less safety against earthquakes than Class A components, but more safety than ordinary components. Structures in this class are the waste disposal building and the turbine building, while the respective equipment and piping system are the reactor auxiliary system, the waste disposal system, and the turbine generator. The design horizontal seismic coefficient (c_h) was equal to one and a half times the yielding at the respective seismic zone of the Japanese Building Code, $c_h = 1.5 \times 0.18 = 0.27$. No vertical seismic actions are taken into consideration.

CLASS C. Includes the items that are not listed in the previous categories. They require ordinary safety against earthquakes, and are designed according to the Japanese Building Code.

In short, the various items in the Fukushima Daiichi NPP are designed with the following maximum seismic coefficients:

- Items in Class As: the maximum spectral values for ground acceleration equal to 0.27g, corresponding to MCE, and for damping ratios between 2 and 5% are 0.61 and 0.40, respectively (see Fig. 5). The vertical seismic coefficient equal to $c_v = 0.09$ is constant along the height of the structure. The same items are designed statically, belonging also to class A. The safer was chosen.
- Items in Class A: horizontal $c_h = 0.54$, vertical $c_v = 0.09$ (static analysis).
- Items in Class B: horizontal $c_h = 0.27$, no vertical seismic loading (static analysis).
- Items in Class C: horizontal $c_h = 0.18$, no vertical seismic loading (static analysis).

The whole design was based on allowable working stresses with an increase for transient (earthquake) loads of 50% for steel and 100% for concrete. Taking into consideration all these facts to achieve a linear-elastic response, without cracks or damage that are essential for the SSD function of the NPP, during the design earthquake, the maximum applied seismic motion must produce accelerations at the respective centers of mass corresponding to no more than 1/2 to 2/3 the applied design seismic coefficients, i.e. 0.27 to 0.40 for items belonging to Class A and As. These values represent, roughly, the earthquake resistance of the installed components at the Fukushima Daiichi NPP plant. Any earthquake motion producing higher values may cause structural problems.

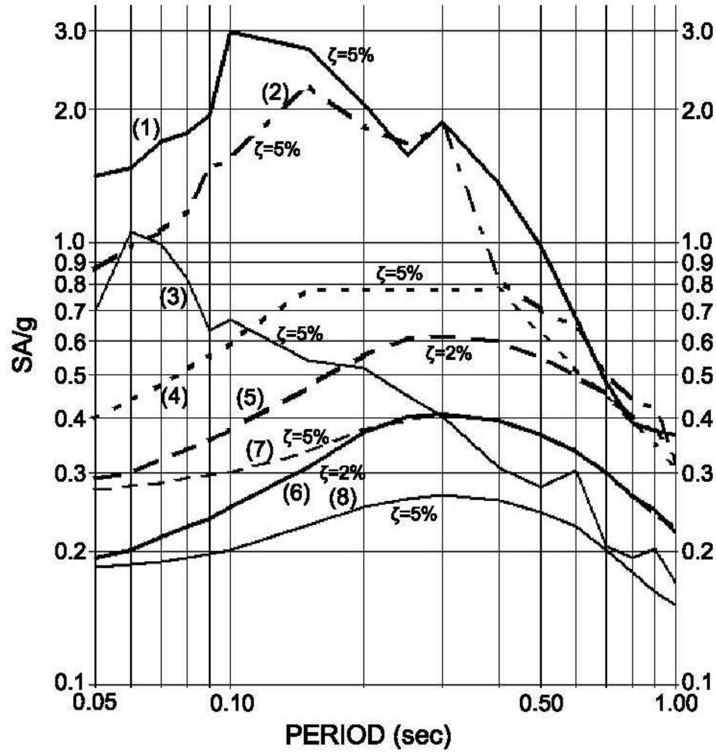

Figure 5. Comparison of design and recorded spectra, with reference to EC-8. (1) Arithmetic mean of N-S components of FKS005 and FKS010 records; (2) Arithmetic mean of E-W components of FKS005 and FKS010 records; (3) Arithmetic mean of U-D components of FKS005 and FKS010 records; (4) Design shape of spectrum according to EC-8 for a_g=0.24g (medium seismic zone in Greece) and highest importance factor 1.3 (valid for Greece); (5), (7) NPP's design spectrum corresponding to MCE earthquake; (6), (8) NPP's design spectrum corresponding to SSD earthquake, for damping ratios (ζ) 2 and 5%, respectively.

To compare Fukushima Daiichi NPP's design spectra with those recorded during the March 11th, 2011 mainshock we identified the recording stations nearest to the plant that are sited on firm soils or soft rock sites. Two K-NET stations qualified under these criteria, namely FKS005 and FKS010 situated approximately 24.3 km north and 21.2 km south of the plant, respectively. According to the response spectra of these two recordings the respective equivalent seismic coefficients developed in the center of gravity of a structure according to its respective natural period from 0.3 to 0.5 sec and 5% damping, see Fig. 5, range from 0.8 to 1.5g. These values for a damping 2% should be increased by about 15-25%, i.e. 1.0 to 1.8g, although due to overstress beyond the yielding level a higher damping than 2% may actually be more realistic. The comparison between the installed seismic resistance and actual earthquake loading gives a ratio of about 1 over 3. More work is needed to better estimate the ground motion at the plant and until the actual main shock recordings and the respective response spectra at the plant will be released.

If we consider also, the vertical earthquake component, only the items belonging to Class A and As have been designed for a vertical seismic coefficient equal to 50% of the horizontal seismic coefficient yielding at the site according to the Japanese Building Code. This is equal to 0.09 which finally is the (1/2) x (1/3) = 1/6 ≈ 17% of the applied horizontal ones instead of the 70% ratio used in recent codes (EC-8, 2004). Following the same logic as the above mentioned, we reach almost to the same ratio (1 over 3) as far as the vertical direction is concerned.

It is interesting to mention that the recorded ground motions, as shown in Fig. 5, present their maximum values in higher frequencies which result to produce high acceleration spectral values in periods shorter than 0.5 sec. This fact is certainly unfavorable for rigid, monolithic structures with rather short vibration periods such as the NPPs. This shape of the design response spectrum was predicted by Prof. G. Housner who proposed it as it is shown in Fig. 5, with maximum values between 0.1 and 0.5 sec.

Since the ground shaking motions (horizontal and vertical) are almost simultaneous, the effect of the vertical component with response spectral values exceeding 1.0g means that the whole original calculations are totally capsized. This is simply because there is no friction, unless it is highly reduced between the structure and the supporting ground.

Additionally to these rough estimations, duration of the strong phase of the ground motion during the March 11 main shock was long and of the order of 40 – 50 sec, which means 30 to 50 strong cycles of loading into the plastic domain of the materials, which can be really exhaustive for the building materials, the foundation ground and the structure. At this point it must be mentioned that a structure once is responding into the plastic domain, its natural periods are elongated becoming more flexible and more susceptible to more distant seismic events. Under this assumption of plastic response, the loading cycles are cumulative.

One of the most vital and seismically vulnerable parts of the plant is its piping system. Pipes are subject to seismic excitation from their supports on the respective structural components. The various structural components during the earthquake must have experienced quite large accelerations. In the stiff structures of the plant, inside the RC cover structure, very strong accelerations, perhaps exceeding acceleration of gravity, may have been developed, as well as large relative displacements. The majority of the structures are like “tube in tube” type, connected to each other with a stabilizer at the top in order to prevent impact effects. Additionally, pipes that are not founded on the ground but are supported on other structures during the strong phase were excited at each of their supports that are generally out of phase among themselves. The differential displacements of their supports might have amplified the pipe responses significantly. The piping system of the plant was equipped with a lot of dampers and elastomeric bellows in critical key positions of the piping network. Therefore, we estimate that no additional dynamic motion could be developed in the piping system itself if those dampers and bellows were well maintained and were adequately functioning during the earthquakes. If these damping mechanisms were not properly functioning, large resonance phenomena in the piping system would develop. Furthermore, rather long duration of these very strong excitations with differential support displacements, may have resulted in producing inelastic and irreversible damage to the piping system of the plant. On the other hand, it must be mentioned that for the piping system the input design accelerations for Class As was around 0.61g while for Class A was 0.54g.

In Figs 6 and 7, two pictures during the construction phase of the Fukushima NPP are presented. By these pictures an advanced level of construction technology is presented as would be expected for such a critical facility.

Figure 6. The base of the steel structure at the early phases of construction of the Fukushima Daiichi NPP, by courtesy Watabe (1970).

Figure 7. The containment vessel of the Fukushima Daiichi NPP almost finished, by courtesy Watabe (1970).

4. CONCLUSIONS

According to our comparative results the Fukushima Daiichi NPP's seismic capacity should have been upgraded to a level of about three times its initially installed capacity in order to withstand safely ground shaking experienced during the March 11th event. It is important to know the type of seismic upgrading that may have taken place at the plant during its 40 years of operation. A most effective type of seismic upgrading would be the application of a base isolation system, seismic energy absorbing devices e.tc. If such adequate upgrading was not performed it is supposed that the Fukushima Daiichi NPP was quite likely severely damaged before the tsunami attack.

A further detailed investigation is needed when the strong motion records at the Fukushima Daiichi NPP will be available.

In critical facilities such as NPPs, the combination of the two design methods static and dynamic, as have been recommended for the Fukushima Daiichi NPP, is towards the safe side. The static method ensures the least necessary requirements, while the dynamic method projects the maximum values.

It is also important to point out that in such facilities it would be perhaps preferable not to classify the various components by importance class. All components must possess the highest level of seismic safety.

During the phase of the conception design of such facilities it must be taken under consideration that always a higher hazard value than the designed one may occur during its functional life. Contemporary earthquake engineering technology disposes several methods for adequately safe structures also under this assumption.

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